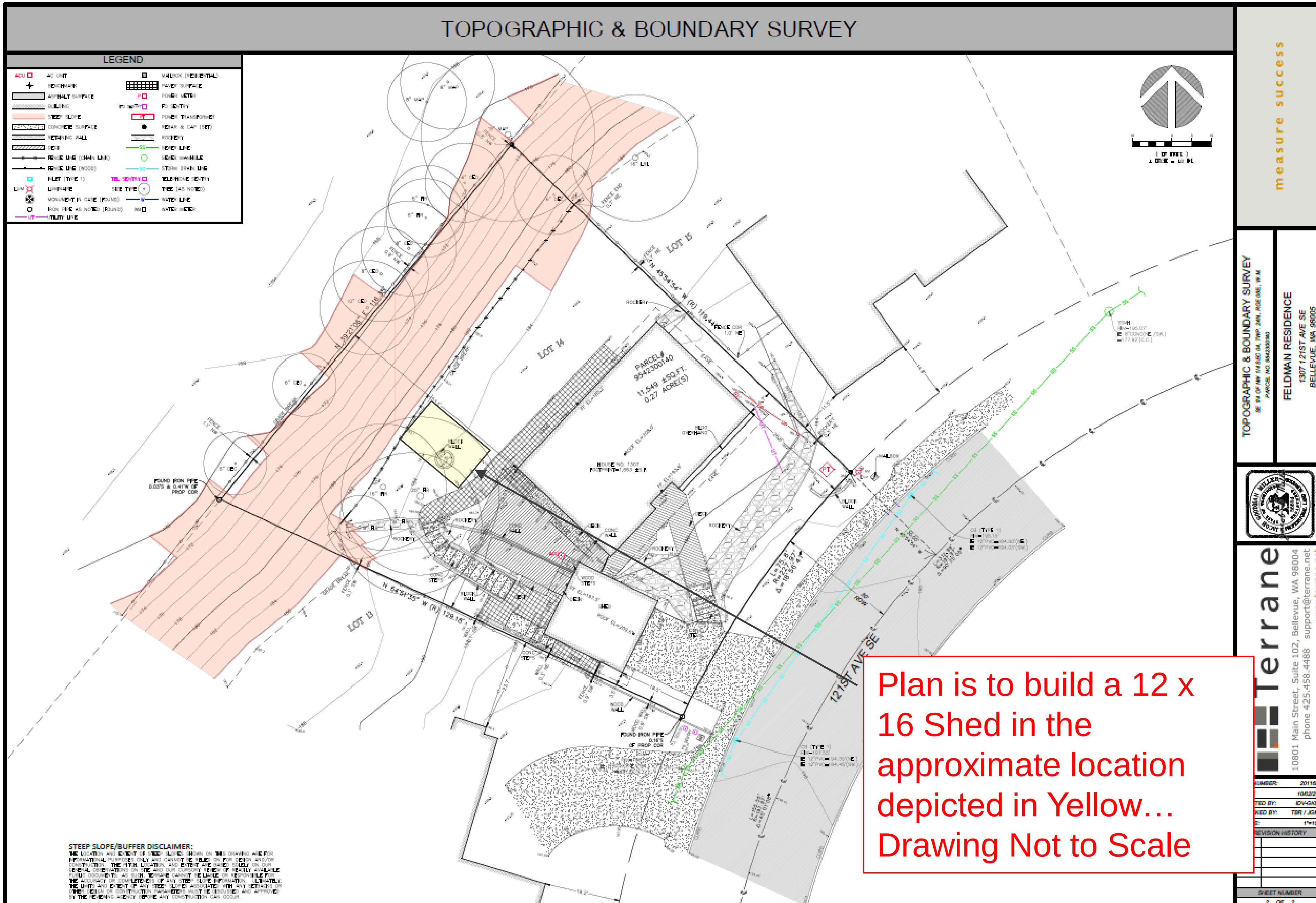
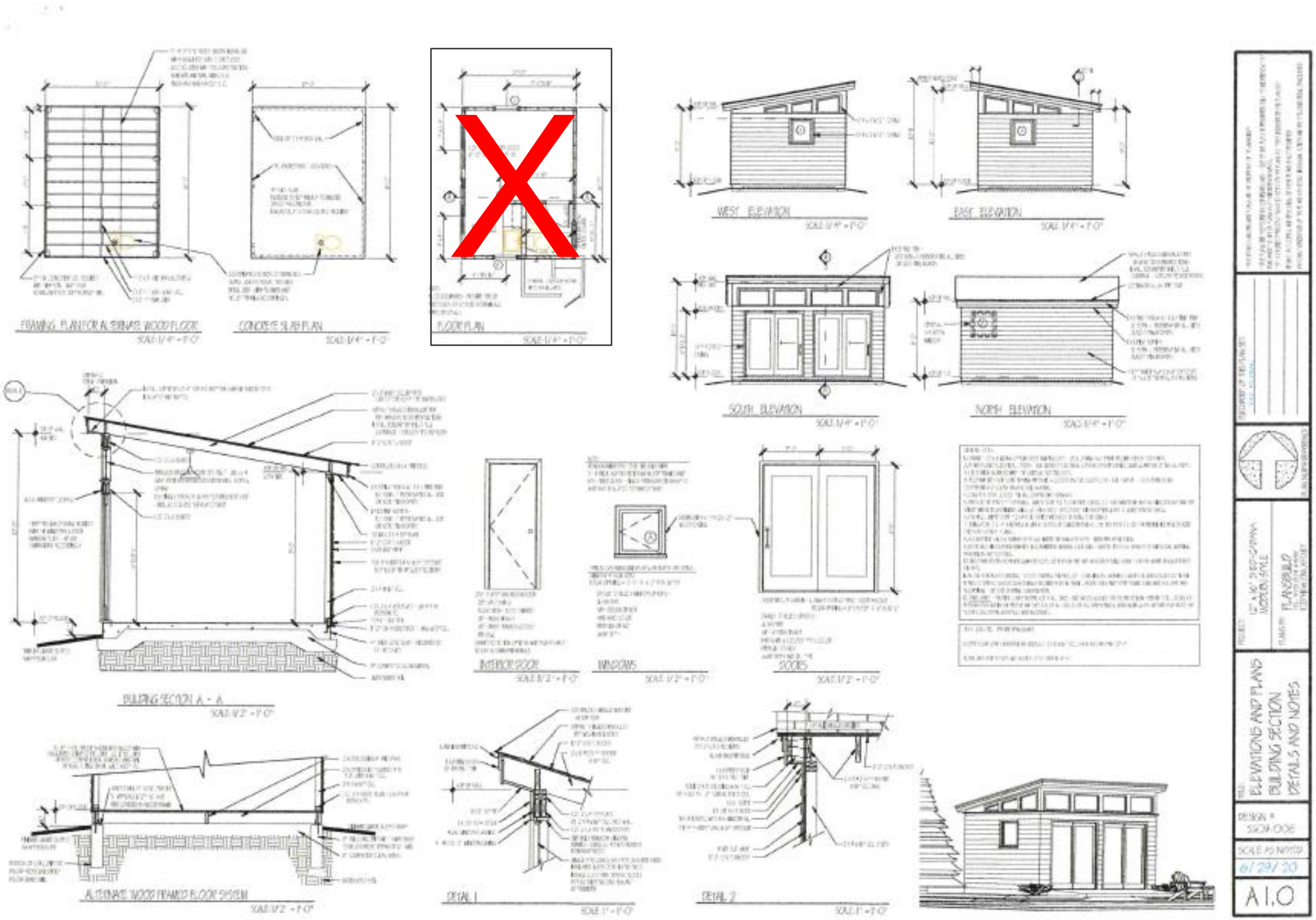
Feldman Shed Site Plan – 1307 121st Ave SE



Plan is to build a 12 x 16 Shed in the approximate location depicted in Yellow… Drawing Not to Scale

Feldman Shed Plan – 1307 121st Ave SE



Feldman Shed Plan – 1307 121st Ave SE

Stock plans I purchase for framing show optional bathroom.

We will not be using those or adding plumbing.

December 17, 2020

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Limited Geotechnical Engineering Report
Proposed Accessory Structure
1307 – 121st Avenue Southeast
Bellevue, Washington
PN: 9542300140
Doc ID: Feldman.121stAveSE.RG

INTRODUCTION

This geotechnical engineering report summarizes our site observations, geotechnical data review, engineering analyses, and provides geotechnical recommendations and design criteria for the proposed shed to be constructed at 1307 – 121st Avenue Southeast in Bellevue, Washington. The approximate site location is shown on the Site Location Map, included as Figure 1.

Our understanding of the project is based on conversations with you; a review of available published geologic literature; our November 23, 2020 site visit and subsurface explorations; our review of the provided *Topographic & Boundary Survey* for the parcel prepared by Terrane dated October 2, 2020; our understanding of the City of Bellevue (the City) building and development codes; and our experience in the area.

The site is currently developed with an existing single-family residence, detached garage, paved driveway, and associated utilities. Based on King County Department of Assessment records, we understand that the existing residence was constructed in 1966. We understand that you propose to construct a shed west of the existing residence, near the top of a slope that is steeper than 40 percent with vertical relief of 12 to 14 feet. Because of the height and steepness of the site slopes and the proximity of the proposed development to those slopes, the City is requiring a critical areas report to address the steep slope per the City of Bellevue Land Use Code chapter 20.25H. Additionally, modifications or reductions of the City's standard 50 feet top-of-slope critical area buffer require a stability analysis in accordance with the City of Bellevue Development Services' "Geotechnical Report Requirements" document. We have prepared this *Limited Geotechnical Engineering Report* to address the City's Critical Areas code and relevant geotechnical report requirements, and to provide foundation recommendations.

SCOPE

The purpose of our services was to evaluate the surface and subsurface conditions reported by others across the site as a basis for providing geotechnical recommendations and design criteria for the proposed development. Specifically, the scope of services for this project included the following:

1. Reviewing the available geologic, hydrogeologic, and geotechnical data for the site area;
2. Exploring surface and subsurface conditions by reconnoitering the site and excavating 2 hand borings at select locations across the site;
3. Describing surface and subsurface conditions, including soil type, depth to groundwater, and an estimate of seasonal high groundwater levels;
4. Addressing the appropriate criteria for geologically hazardous areas per the City of Bellevue Land Use Code chapter 20.25H;
5. Evaluating the global stability of the proposed development using Slide2 by Rocscience;
6. Providing recommendations regarding deep foundation elements, including pin piles, as appropriate;
7. Providing geotechnical conclusions and recommendations regarding site grading activities, including site preparation, subgrade preparation, fill placement criteria, suitability of on-site soils for use as structural fill, temporary cut slopes and drainage and erosion control measures; and
8. Preparing this written *Limited Geotechnical Engineering Report* summarizing our site observations and conclusions, and our geotechnical recommendations and design criteria, along with the supporting data.

The above scope of work was completed in accordance with our *Proposal for Geologic Hazard Assessment* dated November 11, 2020. We received written authorization to proceed with our scope of services from you the same day.

SITE CONDITIONS

Literature Review

In addition to publicly available topographic, geologic, and hydrogeologic information, as a component of this study we reviewed excerpts from the *Hydrogeologic Report on the Proposed Wilburton Tunnel* by Robinson & Roberts dated November 30, 1962. The documents were retrieved from the Washington Department of Natural Resources (DNR) Geologic Information Portal: Subsurface Database.

The “Log of Borings” includes a boring, B48, which appears to have been located about 400 to 500 feet south of the subject parcel of this report. Boring B48 was advanced from elevation 195.9 to 77 feet. The reference datum was not defined and may vary several feet or more from the current NAVD 88 datum. The boring encountered bluish gray and brown silt and clay to approximately elevation 165 feet. Underlying the silt and clay, boring B48 encountered brown sandy gravel grading to brown sand that extended to the full depth explored. The logs identified the entire sequence as advance outwash, with the upper fine-grained deposits labeled as the “silt member of advance outwash”.

Surface Conditions

The project site is located along 121st Avenue Southeast in the Woodridge neighborhood of Bellevue, Washington. The irregularly-shaped parcel measures about 75 to 115 feet wide (southwest to northeast) by 122 to 132 feet long (northwest to southeast) and encompasses about 0.27 acres. The site is bounded by 121st Avenue Southeast to the southeast and by residential development to the northeast, northwest, and southwest.

The site is located along the northwest margin of a glacial upland area that slopes down to the north and west towards Interstate 405 and continues towards the Mercer Slough area. The subject parcel slopes down from 121st Avenue Southeast to the northwest at an average slope of about 15 percent for a horizontal distance of 95 to 100 feet and vertical relief of approximately 14 feet. The ground surface steepens to about 50 percent with vertical relief of 12 to 14 feet to the northwest property boundary. The ground surface flattens approximately along the property boundary to less than 15 percent. The total topographic relief across the subject parcel is on the order of 26 feet. The existing site configuration and topography are shown on the attached Site Vicinity Map, Figure 2. An excerpt of the site survey is shown on the Site & Exploration Map, Figure 3.

The upland portion of the site is generally vegetated with typical residential landscaping and a few mature fir trees. The area where the accessory structure is proposed is currently surfaced with landscaping rock. The steep slope is well-vegetated with ivy, and scattered trees (primarily cedar) along the toe of slope. No evidence of erosion or slope movement was observed at the site at the time of our site visit.

Site Soils

The USDA Natural Resource Conservation Service (NRCS) Web Soil Survey maps the soils in the vicinity of the parcel as Alderwood gravelly loam (AgC) and Kitsap silt loam (KpD). Alderwood gravelly loam (AgC) forms on slopes of 8 to 15 percent, are derived from glacial till, are listed as having a moderate erosion hazard, and are in hydrologic soils group C/D. The Kitsap soils (KpD) are derived from glacial lake sediment, form on slopes of 15 to 30 percent, have a moderate erosion hazard when exposed, and are in hydrologic soils group C. An excerpt of the NRCS soils map is included as Figure 4.

Site Geology

The Washington State Department of Natural Resources (DNR) Geology Portal maps the site area as being underlain by advance outwash deposits (Qga). These glacial soils were deposited during the Vashon Stade of the Fraser Glaciation, approximately 12,000 to 15,000 years ago. The advance outwash soils consist of poorly sorted, lightly stratified mixture of sand and gravel that may contain localized deposits of clay and silt that were deposited by meltwater streams emanating from the advancing ice mass. The advance outwash is considered over-consolidated and exhibits high strength and low compressibility characteristics where undisturbed. No areas of pre-historic landslides or mass wasting are shown on the map within the site vicinity (within 300 feet of the site). An excerpt of the above referenced geologic map is attached as Figure 5.

We also reviewed the Washington DNR Landslide Inventory, which maps two areas of historic landslide on the eastern slopes of the Woodridge highland area. The mapped historic landslide areas are approximately 800 and 1,700 feet horizontally from the subject parcel. No mapped areas of landslide deposits are mapped within 300 feet of the subject parcel. An excerpt of the Washington DNR Landslide Inventory Map for the site area are included as Figure 6.

Subsurface Explorations

On November 23, 2020, field representatives from GeoResources, LLC (GeoResources) visited the site and advanced two hand boring test holes to depths of 5½ and 13½ feet below the existing ground surface. Table 1, below, summarizes the approximate functional locations, surface elevations, and termination depths of the explorations.

TABLE 1:
APPROXIMATE LOCATIONS, ELEVATIONS AND DEPTHS OF EXPLORATIONS

Boring Number	Functional Location	Estimated Surface Elevation[1] (feet)	Termination Depth (feet)	Termination Elevation (feet)
HB-1	Toe of slope	170	5½	164½
HB-2	Top of slope	183	13½	169½

Notes:
1 = Surface elevation interpolated from contours on the provided site survey (datum NAVD 88)

The specific number, locations, and depths of our explorations were selected based on our understanding of the proposed development and were adjusted in the field based on consideration for underground utilities, existing site conditions, site access limitations and encountered stratigraphy. Field representatives from our office completed logs of the subsurface conditions encountered, obtained representative soil samples, and observed pertinent site features. Representative soil samples obtained from the explorations were placed in sealed plastic bags and taken to our laboratory for further examination and testing as deemed necessary.

Relatively disturbed, but representative, soil samples were obtained at selected depths using portable Porter soil sampling equipment. The hand-operated equipment consists of a 1.4-inch outside-diameter (1.0-inch inside-diameter) split-spoon sampler connected to extension rods of the same diameter as the barrel. The Porter sampling method consists of driving the sampler 18-inches into the soil with a 45-pound weight, with a drop of 18 inches. The number of blows required to drive the sampler through each 6-inch interval is counted, and the total number of blows struck during the final 12 inches is recorded as the Porter Penetration Test (PPT). If a total of 50 blows are recorded within any 6-inch interval (refusal), the driving is stopped, and the blow counts are recorded as 50 blows for the actual distance the sampler was driven. The resulting PPT values indicate the relative density of granular soils and the consistency of cohesive soils and the energy and size of the test are correlated to approximately match the values that would be obtained from a Standard Penetration Test.

The explorations advanced as part of this evaluation indicate the subsurface condition at specific locations only, as actual subsurface conditions can vary across the site. Furthermore, the nature and extent of such variation would not become evident until additional explorations are performed or until construction activities have begun. Based on our experience in the area, it is our opinion that the soils encountered in the exploration are generally representative of the soils at the site. The soils encountered were visually classified in accordance with the Unified Soil Classification System (USCS) and ASTM D: 2488. The USCS is included in Appendix A as Figure A-1. The approximate locations of our hand borings are shown on the attached Site & Exploration Plan Figure 2, while the descriptive logs of our explorations are included in Appendix A as Figures A-2 and A-3.

Subsurface Conditions

At the locations of our hand borings we encountered variable subsurface conditions that, in our opinion, partially differ from the mapped stratigraphy. Topsoil thicknesses at the locations explored were about 6 to 12 inches. Our hand boring at the toe of slope (HB-1) encountered soils we interpret as native advance outwash, while our hand boring at the top of slope (HB-2) encountered about 12 feet of fill material overlying the advance outwash. Table 2, below, summarizes the approximate thicknesses, depths, and elevations of selected soil layers.

Fill

Underlying the topsoil, hand boring HB-2 encountered about 1½ feet of tan silty sand in a medium dense, moist to wet condition. The silty sand mantled 10 feet of tan silt with variable sand with clasts of hard gray clayey silt in a stiff, moist condition. Small plastic debris was encountered in the tan silt at about 9 feet below ground surface. We interpret the tan silty sand and tan silt to be consistent with fill material.

Advance Outwash

Underlying the topsoil, hand boring HB-1 encountered about 3 feet of brown to gray sandy silt in a stiff, moist condition that we interpret to be weathered advance outwash. Underlying the sandy silt in HB-1 and the undocumented fill in hand boring HB-2, we encountered tan to brown silty gravel with sand in a medium dense to very dense, moist condition to the full depth explored. We interpret the silty gravel with sand to be consistent with advance outwash deposits.

TABLE 2:
APPROXIMATE THICKNESS, DEPTHS, AND ELEVATION OF SOIL TYPES ENCOUNTERED IN EXPLORATIONS

Exploration Number	Thickness of Topsoil (feet)	Thickness of Fill (feet)	Thickness of Silty Advance Outwash (feet)	Depth to Advance Outwash (feet)	Elevation of Top of Advance Outwash (feet)
HB-1	1	NE	3	4	165½
HB-2	½	11½	NE	12	171

Notes:
Elevation interpolated from contours on the provided site survey (datum NAVD 88)
NE = not encountered

Laboratory Testing

Geotechnical laboratory tests were performed on a select sample retrieved from hand boring HB-1 to estimate index engineering properties of the soil encountered. Laboratory testing included visual soil classification per ASTM D2488 and ASTM D2487, moisture content determination per ASTM D2216, and grain size analysis per ASTM D6913 standard procedures. The results of the laboratory tests are included in Appendix B and summarized in Table 3.

TABLE 3:
LABORATORY TEST RESULTS FOR ON-SITE SOILS

Sample	Lab ID Number	Soil Type	Gravel Content (percent)	Sand Content (percent)	Silt/Clay Content (percent)	D10 (mm)
HB-2, S-5, D: 12.5’	099841	GM	40.6	35.6	23.8	< 0.074

Groundwater Conditions

No groundwater was observed in our explorations at the time of excavation. The upper silt soils encountered in our hand borings were in a moist to wet condition. These soils appeared to be poorly drained soils and we anticipate that perched groundwater may develop in the upper several feet of the silt during periods of extended precipitation.

Regional groundwater was recorded at elevation 92.2 feet in boring B48 in the 1962 *Hydrogeologic Report on the Proposed Wilburton Tunnel*. Elevation 92.2 feet is approximately 78 feet below the toe of the steep slope on the subject parcel. As stated, we anticipate the vertical datums used in the 1962 boring and the current survey differ by several feet, however the recorded groundwater level would still be several tens of feet below the ground surface on the subject parcel. We anticipate fluctuations in the local groundwater levels will occur in response to precipitation patterns, off-site construction activities, and site utilization.

ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our data review, site reconnaissance, subsurface explorations and our experience in the area, it is our opinion that the site is suitable for the proposed accessory structure provided structural setbacks from the top of slope are met. We have provided recommendations for addressing the required setback by using pin piles. Pertinent conclusions and geotechnical recommendations regarding the design and construction of the proposed structure are presented below.

Landslide Hazard Area - per City of Bellevue LUC 20.25H.120

The City of Bellevue defines a landslide hazards as areas of slopes of 15 percent or more with more than 10 feet of rise, which also display any of the following characteristics:

A. Areas of historic failures, including those areas designated as quaternary slumps, earthflows, mudflows, or landslides.
B. Areas that have shown movement during the Holocene Epoch (past 13,500 years) or that are underlain by landslide deposits.
C. Slopes that are parallel or subparallel to planes of weakness in subsurface materials.
D. Slopes exhibiting geomorphological features indicative of past failures, such as hummocky ground and back-rotated benches on slopes.
E. Areas with seeps indicating a shallow ground water table on or adjacent to the slope face.

F. Areas of potential instability because of rapid stream incision, stream bank erosion, and undercutting by wave action.

No areas of historic failures or landslide deposits are mapped within 300 feet of the subject parcel. The subsurface conditions appear to consist of a less permeable soils (silt) overlying more permeable soils (sand and gravel) that we would not interpret as a plane of weakness or an adverse geologic contact. No areas of hummocky ground, back-rotated benches, or other indicators were observed in the site vicinity. No seepage was observed along the face of the slope. The site is not susceptible to rapid stream incision, stream bank erosion, or undercutting by wave erosion.

The parcel has none of the above listed indicators for landslide hazard and, in our opinion, should not be classified as a landslide hazard area.

Steep Slope Hazard Area - per City of Bellevue LUC 20.25H.120

The City of Bellevue defines a steep slope hazard area as any slope of 40 percent or more that has a rise of at least 10 feet and exceeds 1,000 square feet in area. As shown on the provided topographical survey, the slope in the northwest portion of the site is steeper than 40 percent and has a rise of 12 to 14 feet, and therefore meets the definition of a steep slope hazard area.

Seismic Design

Based on our observations and the subsurface units mapped at the site, we interpret the structural site conditions to correspond to a seismic Site Class "D" in accordance with the 2015 International Building Code (IBC) documents and ASCE 7-10, Chapter 20, Table 20.3-1. This is based on the recorded and assumed range of Standard Penetration Test (SPT) blow counts from the subsurface explorations at the site and mapped geology. These conditions were assumed to be representative for the subsurface conditions for the site in general.

For design of seismic structures using the 2015 IBC, mapped short-period and 1-second period spectral accelerations, SS and S1, respectively, are required. The U.S. Geological Survey (USGS) completed probabilistic seismic hazard analyses (PSHA) for the entire country in November 1996, which were updated and republished in 2002 and 2008. The PSHA ground motion results were obtained from the *ATC Hazard by Location* website. The results of the updated USGS PSHA were referenced to determine S_S and S_1 for this site. The results are summarized in the following table with the relevant parameters necessary for 2015 IBC design.

TABLE 4:
2015 IBC PARAMETERS FOR DESIGN OF SEISMIC STRUCTURES

Spectral Response Acceleration (SRA) and Site Coefficients	Short Period	1 Second Period
Mapped SRA	S_s = 1.329	S_1 = 0.511
Site Coefficients (Site Class D)	F_a = 1.000	F_v =1.500
Maximum Considered Earthquake SRA	S_{MS} = 1.329	S_{M1} = 0.767
Design SRA	S_{DS} = 0.886	S_{D1} = 0.511

Peak Ground Acceleration

The mapped peak ground accelerations (PGA) for this site are 0.576g for the 2,475 year return period and 0.284g for the 475 year return period using the 2014 USGS seismic maps. To account for site class, the PGA is multiplied by a site amplification factor (F_{PGA}) of 1.1 and 1.32, respectively. The resulting site modified peak ground accelerations (PGA_M) are 0.633g for the 2,475 year return period and 0.375g for the 475 year return period. In general, estimating seismic earth pressures (k_h) by the Mononobe-Okabe method or seismic inputs for slope stability analysis are taken as 33 to 50 percent of the PGA_M.

Earthquake-Induced Geologic Hazards

Earthquake-induced geologic hazards may include liquefaction, lateral spreading, slope instability, and ground surface fault rupture. Liquefaction is a phenomenon where there is a reduction or complete loss of soil strength due to an increase in pore water pressure. The increase in pore water pressure is induced by seismic vibrations. Liquefaction mainly affects geologically recent deposits of loose, fine-grained sands that are below the groundwater table. Based on the gradation of the fill and density of the native outwash underlying the site, it is our opinion that the risk for liquefaction to occur at this site during an earthquake is negligible.

In our opinion, the potential for deep-seated slope instability or lateral spreading are also low because of the glacially consolidated advance outwash deposits underlying the site. In addition, the site is not close to major nearby faults (approximately 0.9 miles north of a mapped fault in the Seattle fault zone) and no evidence of ground fault rupture was observed in the subsurface explorations or our site reconnaissance. Therefore, in our opinion the potential for ground surface fault rupture is also low.

Provided the design criteria listed below are followed, the proposed structure should have no greater seismic risk damage than other appropriately designed structures in the Puget Sound area. Additional discussion of the dynamic global stability at the site is included in the "**Slope Stability Analysis**" section, below.

Slope Stability Analysis

We analyzed the global and internal slope stability of the existing and proposed slope geometries using subsurface profile A-A', as shown on Figure 3. Subsurface profile A-A' was selected as the most critical given the location of the proposed development. The slope stability results for the existing and proposed configurations are included as Appendix C.

Per the City of Bellevue Development Services' *Geotechnical Report Requirements* (February 27, 2019), the acceleration factor for pseudo-static seismic analyses "must be based on a peak ground acceleration with a 10 percent probability of exceedance in 50 years (i.e. a 475-year return period)". Using the procedures described in the 2019 WSDOT *Geotechnical Design Manual*, Section 6-3.5, the site modified peak ground acceleration (PGA_M) for the 475-year return period is 0.375g. We applied a seismic acceleration in our slope stability analysis equal to ½ of the PGA_M, or 0.188g.

We used the computer program Slide2, from RocScience, 2020, to perform the slope stability analyses. The computer program Slide2 uses a number of methods to estimate the factor of safety (FS) of the stability of a slope by analyzing the shear and normal forces acting on a series of vertical "slices" that comprise a failure surface. Each vertical slice is treated as a rigid body; therefore, the forces and/or moments acting on each slice are assumed to satisfy static equilibrium (i.e., a limit equilibrium analysis). The FS is defined as the ratio of the forces available to resist movement to the forces of the driving mass. A FS of 1.0 means that the driving and resisting forces are equal; an FS less than 1.0 indicates that the driving forces are greater than the resisting forces (indicating failure). We used the Generalized Limit Equilibrium method using the Morgenstern-Price analysis, which satisfies both moment and force equilibrium, to search for the location of the most critical failure surfaces and their corresponding FS. The most critical surfaces are those with the lowest FS for a given loading condition, and are therefore the most likely to move.

Soils at the site were modeled using Mohr-Coulomb strength properties. Table 5, below, summarizes the estimated soil parameters used in our stability analyses. In our opinion, based on the blow counts recorded in our hand borings, the assumed values below are appropriate and, in some instances, relatively conservative.

TABLE 5:
MOHR-COULOMB STRENGTH PARAMETERS

Soil Type	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Fill (ML, medium stiff to stiff)	115	22	150
Advance outwash (GM, medium dense to dense)	135	38	0

Profile A-A'

Based on our analyses, the current and proposed site configurations are stable and have the factors of safety shown in Table 6, below. The proposed site configuration includes founding the accessory structure on deep foundation elements extending through the undocumented fill, with bottom elevations 12 feet below ground surface. We anticipate that minimal to no grading will be required for the proposed accessory structure and therefore the overall configuration of site grades should not be significantly changed by the proposed accessory structure.

TABLE 6:
GLOBAL STABILITY ANALYSES RESULTS

Cross Section	Condition	Loading Condition	Factor of Safety
A-A'	Existing	Static	1.7
		Dynamic	1.2
	Proposed	Static	2.0
		Dynamic	1.4

The most likely predicted failure slopes for all conditions are relatively shallow failures along the undocumented fill slope. We interpret the risk of deep-seated slope failure at the site as low.

Recommended Buffer and Setback

Buffers and setbacks are typically used to protect critical areas from disturbance and also to protect the proposed development from damage due to the potential hazard. The following discussions regarding critical area buffers and structure setbacks are based on City of Bellevue Land Use Code 20.25H and International Building Code (IBC) 1808.7, respectively.

Vegetated Buffers

Buffers typically consist of an undisturbed area of native vegetation, retained or established, that extend from the edge of the critical area or hazard. The width of the buffer should be based on the potential hazard and associated risks. Buffer widths are generally measured from the edge of the critical area being protected, in this case top of slope.

Per the City code, a minimum buffer must be established 50 feet from the top of steep slope hazard areas. Existing native vegetation within the buffer area shall be maintained. The buffer may be modified per LUC 20.25H.145. Based on our stability analyses, the steep slope appears to be stable. Provided our foundation recommendations are incorporated into the project plans, the potential hazard and associated risks from the slope are very low. As previously stated, the slope appears to be a constructed fill slope and the area of proposed development is currently surfaced with landscaping rock. We recommend that no vegetated buffer be required from the top of the slope.

Building Setbacks

The 2015 International Building Code (IBC), Section 1808.7 requires a building setback from slopes that are steeper than 3H:1V (Horizontal: Vertical) or 33 percent with greater than 10 feet in vertical height, unless evaluated and reduced and/or a structural setback is provided by a licensed geotechnical engineer. The setback distance is calculated based on the vertical height of the slope. The typical 2015 IBC setback from the top of the slope equals one third the height of the slope or 40 feet, whichever is less. The IBC setback from the top of the 14 feet tall slope would be 5 feet.

Where those setbacks cannot be met, a "structural setback" may be used. A structural setback consists of deep foundation elements that, when measured horizontally form the front of the foundation to the face of the slope, meet the minimum slope setback. Recommendations for deep foundation elements can be found later in the report. Because of the undocumented fill underlying the slope, we recommend that foundation elements be extended to a depth of at least 12 feet below ground surface, which would result in a structural setback of approximately 22 feet. This distance is shown on the figures included in Appendix B.

If the recommendations contained herein are included in the design and construction of the proposed residences, the impact of the proposed accessory structure on the slope and adjacent parcels should be minimal.

Foundation Support

We recommend the proposed accessory structure be founded on deep foundation elements based on the depth of undocumented fill and proximity to the steep slope. Because of site access limitations, we anticipate the foundation elements may need to be installed using man-portable equipment. Recommendations are provided below for steel pin piles and helical anchors. Regardless of the option selected, we recommended the foundation elements be driven/screwed to a depth of at least 12 feet and then to refusal.

Deep Foundation Alternative 1 - Pin Piles

Pin piles consist of small diameter Schedule-40 or -80 steel pipe that are driven into the underlying soils to refusal and/or minimum depths required to meet setback criteria. The steel pipe diameters range from 2 to 6 inches. Individual pipe segments typically range from about 5 to 10 feet long and are successively joined with external threaded couplings, internal slip couplings, or butt welded as pile driving progresses. The larger diameter piles use a pneumatic or hydraulic hammer mounted on the arm of a construction vehicle. 2 inch pin piling is recommended where access and height restrictions limit the use of construction machinery. The pin piles have little to no lateral strength, unless battered. The pin piles must obtain adequate embedment to provide support to the structure.

Regardless of diameter or installation method, we recommend that each pin pile be driven to a minimum depth of at least 12 feet below the existing ground surface and then to refusal once the minimum embedment has been reached. Because refusal depths are difficult to predict and because subsurface conditions could vary significantly across the site, a test pile could be installed. The contractor should be prepared for variable pile lengths. Also, it may be necessary to modify pile layouts if rocks or other obstructions are encountered during pile-driving. While pin piles can be over-driven (beyond refusal criteria) to the minimum embedment depth, tracked machinery with a larger hammer is typically required.

When refusal and minimum embedment have been achieved, the pin piles can be cut to a predetermined height or elevation. To provide a good bond between the piles and the existing foundation, a steel bracket is typically installed on the foundation element, with an adjustable element to provide a pre-loaded condition. A structural engineer should be responsible for designing the reinforced steel and foundation elements. The minimum pile spacing (center to center) shall be determined by the structural engineer.

For the proposed accessory structure, we recommend that 2 inch pin piling be utilized because of access restrictions. A properly installed pin pile driven to refusal (defined by the required capacity, installation contractor, and/or accepted construction practice) should provide the following allowable axial capacities. These capacities assume a minimum pile spacing (center to center) of six diameters, and a maximum length/diameter ratio of 180.

TABLE 7:
PIN PILES ALLOWABLE VALUE

	2-inch diameter
Static Compressive Capacity	4,000 pounds
Transient Compressive Capacity	5,300 pounds

Deep Foundation Alternative 2 - Helical Anchors

Helical anchors (such as the proprietary systems offered by AB Chance and Atlas Systems) typically consist of a square or circular shaft (1.5-inch square is typical) with an 8 to 12 inch diameter helix located at the leading edge. The helix is rotated and is advanced into the soil like a screw, similar to soil augers commonly used for drilling. The anchors can be used either in tension or compression and either as underpinning for foundations or tiebacks for walls. Depending on the capacity required, one or more helices may be located along the shaft, typically at about 1- to 3-foot intervals. The smallest helix is typically at the tip of the anchor, with the sizes becoming progressively larger.

Helical anchors are screwed into the ground with rotary-type torque motor until refusal conditions are met. Refusal is typically defined as achieving a specific torque that corresponds to a specific compressive capacity. After the anchor has been installed it is typically attached to the structural member it will support using an off the shelf bracket and tightened to remove the slack from the system.

The lateral capacity of the battered piling may be taken as the horizontal vector of the axial pile capacity. The capacity of the anchor in uplift (tension) is related to the torsion resistance encountered as the anchor is installed. Torque monitoring of individual anchors is completed as the anchors are installed using a shear pin that shears once the design load (torque) is achieved.

Design loads between 10 and 30 kips are typical, depending on the configuration of the helices and the soil conditions. Anchors should be spaced a minimum distance of three times the largest helix diameter measured from the edges of the helices.

The torque required to achieve the design load is typically provided by the anchor manufacturer based on their testing data. Load testing should be performed in accordance with the Quick Load Test Method (ASTM D1143) on 3 percent, or a minimum of 1, of the installed helical anchors.

Temporary Excavations

All job site safety issues and precautions are the responsibility of the contractor providing services/work. The following cut/fill slope guidelines are provided for planning purposes only. We do not anticipate temporary cut slopes will be necessary during grading operations, but may be required for utility installation.

All excavations at the site associated with confined spaces, such as utility trenches and retaining walls, must be completed in accordance with local, state, or federal requirements. Based on current Washington Industrial Safety and Health Act (WISHA, WAC 296-155-66401) regulations, we classify the fill material as a Type C soil and the advance outwash deposits as a Type B soil.

According to WISHA, for temporary excavations of less than 20 feet in depth, the side slopes in Type B soils should be laid back at an inclination of 1H:1V or flatter from the toe to the top of the slope, while Type C soils should be laid back at a slope inclination of 1.5H:1V or flatter from the toe to top of the slope. It should be recognized that slopes of this nature do ravel and require occasional maintenance. All exposed slope faces should be covered with a durable reinforced plastic membrane, jute matting, or other erosion control mats during construction to prevent slope raveling and rutting during periods of precipitation. These guidelines assume that all surface loads are kept at a minimum distance of at least one half the depth of the cut away from the top of the slope and that significant seepage is not present on the slope face. Flatter cut slopes will be necessary where significant raveling or seepage occurs, or if construction materials will be stockpiled along the top of the slope.

Where it is not feasible to slope the site soils back at these inclinations, a retaining structure should be considered. Where retaining structures are greater than 4 feet in height (bottom of footing to top of structure) or have slopes of greater than 15 percent above them, they should be engineered per Washington Administrative Code (WAC 51-16-080 item 5). This information is provided solely for the benefit of the owner and other design consultants, and should not be construed to imply that GeoResources assumes responsibility for job site safety. It is understood that job site safety is the sole responsibility of the project contractor.

Site Drainage

All ground surfaces, pavements and sidewalks at the site should be sloped away from the structures. Surface water runoff should be controlled by a system of curbs, berms, drainage swales, and or catch basins, and conveyed to an appropriate discharge point. Collected water should not be directed onto the steep slope.

EARTHWORK RECOMMENDATIONS

Site Preparation

All structural areas on the site to be graded should be stripped of vegetation, organic surface soils, and other deleterious materials including existing structures, foundations or abandoned utility lines. The proposed location of the accessory structure was surfaced with landscaping rock at the time of our site visit. We anticipate that minimal to no stripping will be required for the proposed development.

Where placement of fill material is required, the stripped/exposed subgrade areas should be compacted to a firm and unyielding surface prior to placement of any fill. Excavations for debris removal should be backfilled with structural fill compacted to the densities described in the "**Structural Fill**" section of this report.

The exposed subgrade soil should be probed with a ½-inch diameter steel T-probe. Soft, loose, or otherwise unsuitable areas delineated during probing should be recompacted, if practical, or over-excavated and replaced with structural fill. The depth and extent of overexcavation should be evaluated by our field representative at the time of construction. The areas of old fill material should be evaluated during grading operations to determine if they need mitigation, recompaction, or removal.

Structural Fill

All material placed as fill associated under building areas should be placed as structural fill. The structural fill should be placed in horizontal lifts of appropriate thickness to allow adequate and uniform compaction of each lift. In general, where hand operated equipment such as a jumping jack or plate compactor will be used, loose lift thickness should not exceed about 6 inches. Structural fill should be compacted to at least 95 percent of MDD (maximum dry density as determined in accordance with ASTM D: 1557).

The appropriate lift thickness will depend on the structural fill characteristics and compaction equipment used. We recommend that the appropriate lift thickness be evaluated by our field representative during construction. We recommend that our representative be present during site grading activities to observe the work and perform field density tests.

The suitability of material for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines (material passing US No. 200 sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. During wet weather, we recommend use of well-graded sand and gravel with less than 5 percent (by weight) passing the US No. 200 sieve based on that fraction passing the 3/4-inch sieve, such as *Gravel Backfill for Walls* (WSDOT 9-03.12(2)). If prolonged dry weather prevails during the earthwork and foundation installation phase of construction, higher fines content (up to 10 to 12 percent) may be acceptable.

Material placed for structural fill should be free of debris, organic matter, trash and cobbles greater than 6-inches in diameter. The moisture content of the fill material should be adjusted as necessary for proper compaction.

Suitability of On-Site Materials as Fill

During dry weather construction, any non-organic on-site soil may be considered for use as structural fill, provided it meets the criteria described above in the "**Structural Fill**" section and can be compacted as recommended. If the soil material is over optimum moisture content at the time of excavation, it will be necessary to aerate or dry the soil prior to placement as structural fill. The upper portion of the undocumented fill appeared to be above optimum moisture at the time of our subsurface explorations.

The silty sand to silt fill encountered in hand boring HB-2 has a high fines content and will be very difficult or impossible to compact if over-optimum moisture. The existing fill may be suitable for reuse as structural fill, provided the moisture content is maintained within 2 percent of the optimum moisture level and is compacted in accordance with the "**Structural Fill**" section of this report

We recommend that completed graded-areas be protected prior to wet weather conditions. The graded areas may be protected by a layer of free-draining material such as pit run sand and gravel or clean crushed rock material containing less than 5 percent fines, or covered with plastic.

Erosion Control

Weathering, erosion and the resulting surficial sloughing and shallow land sliding are natural processes. As noted, no evidence of surficial raveling or sloughing was observed at the site. To manage and reduce the potential for these natural processes, we recommend erosion protection measures will need to be in place prior to construction activity on the site. Erosion hazards can be mitigated by applying BMPs outlined in the 2012 *Stormwater Management Manual for Western Washington.*

LIMITATIONS

We have prepared this report for use by Kyle Feldman and other members of the design team, for use in the design of a portion of this project. The data used in preparing this report and this report should be provided to prospective contractors for their bidding or estimating purposes only. Our report, conclusions and interpretations are based on our site reconnaissance, subsurface explorations, and data from others, and should not be construed as a warranty of the subsurface conditions.

Variations in subsurface conditions are possible between the explorations and may also occur with time. A contingency for unanticipated conditions should be included in the budget and schedule. Sufficient monitoring, testing and consultation should be provided by our firm during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork and foundation installation activities comply with contract plans and specifications.

The scope of our services does not include services related to environmental remediation and construction safety precautions. Our recommendations are not intended to direct the contractor's methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design.

If there are any changes in the loads, grades, locations, configurations or type of facilities to be constructed, the conclusions and recommendations presented in this report may not be fully applicable. If such changes are made, we should be given the opportunity to review our recommendations and provide written modifications or verifications, as appropriate.

◆ ◆ ◆

We have appreciated the opportunity to be of service to you on this project. If you have any questions or comments, please do not hesitate to call at your earliest convenience.

Respectfully submitted,
GeoResources, LLC

Neil A. Ferguson, PE
Project Geotechnical Engineer

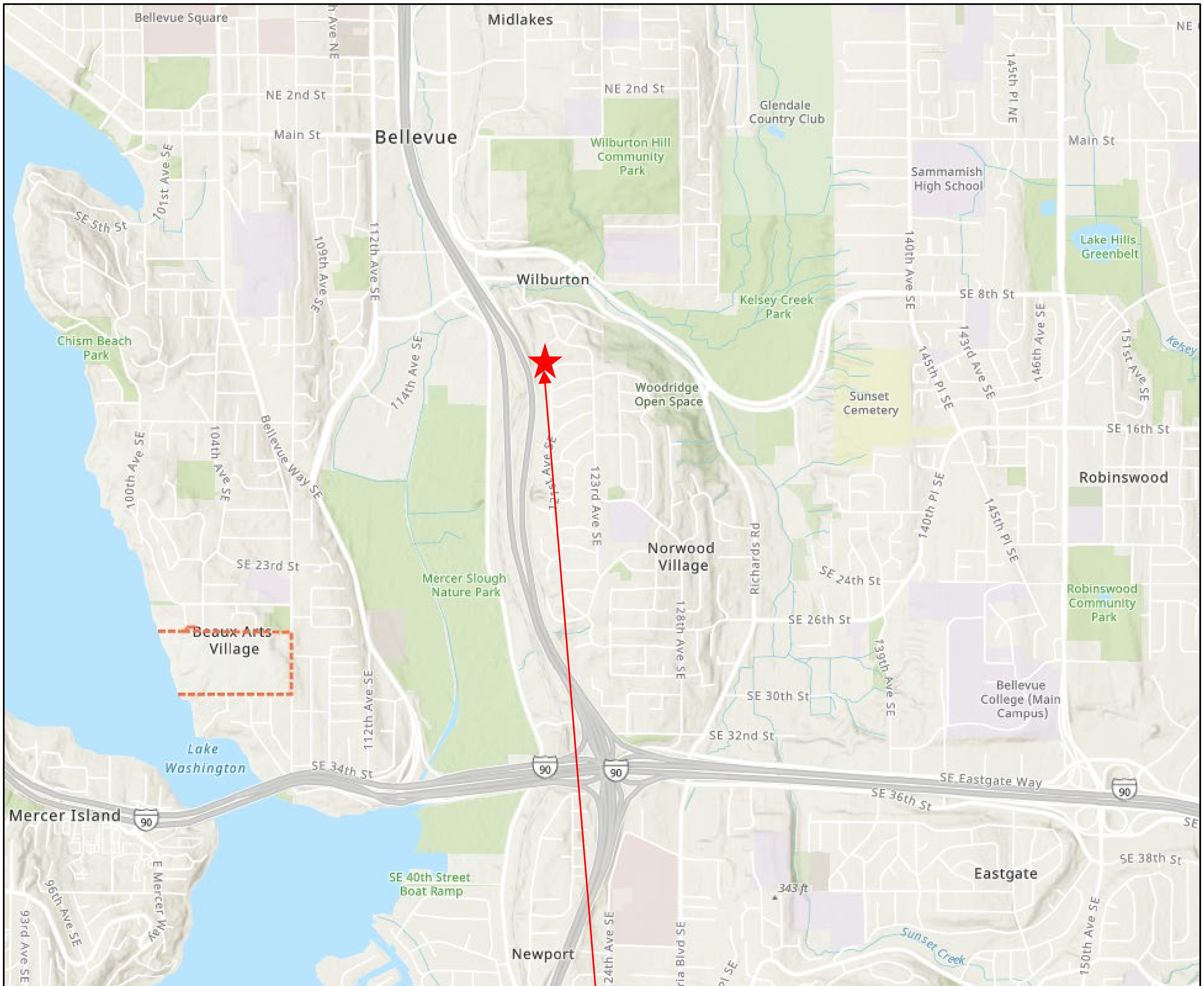
State of Washington
Engineering Geologist
3037
Licensed Geologist
Seth Taylor Mattos
12/17/2020

Seth T. Mattos, LEG
Senior Geologist

ERIC WILLIAM HELLER
STATE OF WASHINGTON
43471
REGISTERED
PROFESSIONAL ENGINEER
12/17/2020

Eric W. Heller, PE, LG
Senior Geotechnical Engineer

NAF:EWH:STM/naf
DocID: Feldman.121stAveSE.RG
Attachments: Figure 1: Site Location Map
Figure 2: Site Vicinity Map
Figure 3: Site & Exploration Plan
Figure 4: NRCS Soils Map
Figure 5: Geologic Map
Figure 6: WA DNR Landslide Inventory Map
Appendix A – Subsurface Explorations
Appendix B – Laboratory Test Results
Appendix C – Slope Stability Analyses



Approximate Site Location

Figure created from Bellevue Map Viewer website

N
W E
S

Not to Scale

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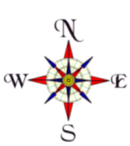
Site Location Map

Proposed Accessory Structure
1307 – 121st Avenue Southeast
Bellevue, Washington
PN: 9542300140

DocID: Feldman.121stAveSE.F	December 2020	Figure 1

Approximate Site Location
Figure created from Bellevue Map Viewer website

Approximate location and number of hand boring



Not to Scale

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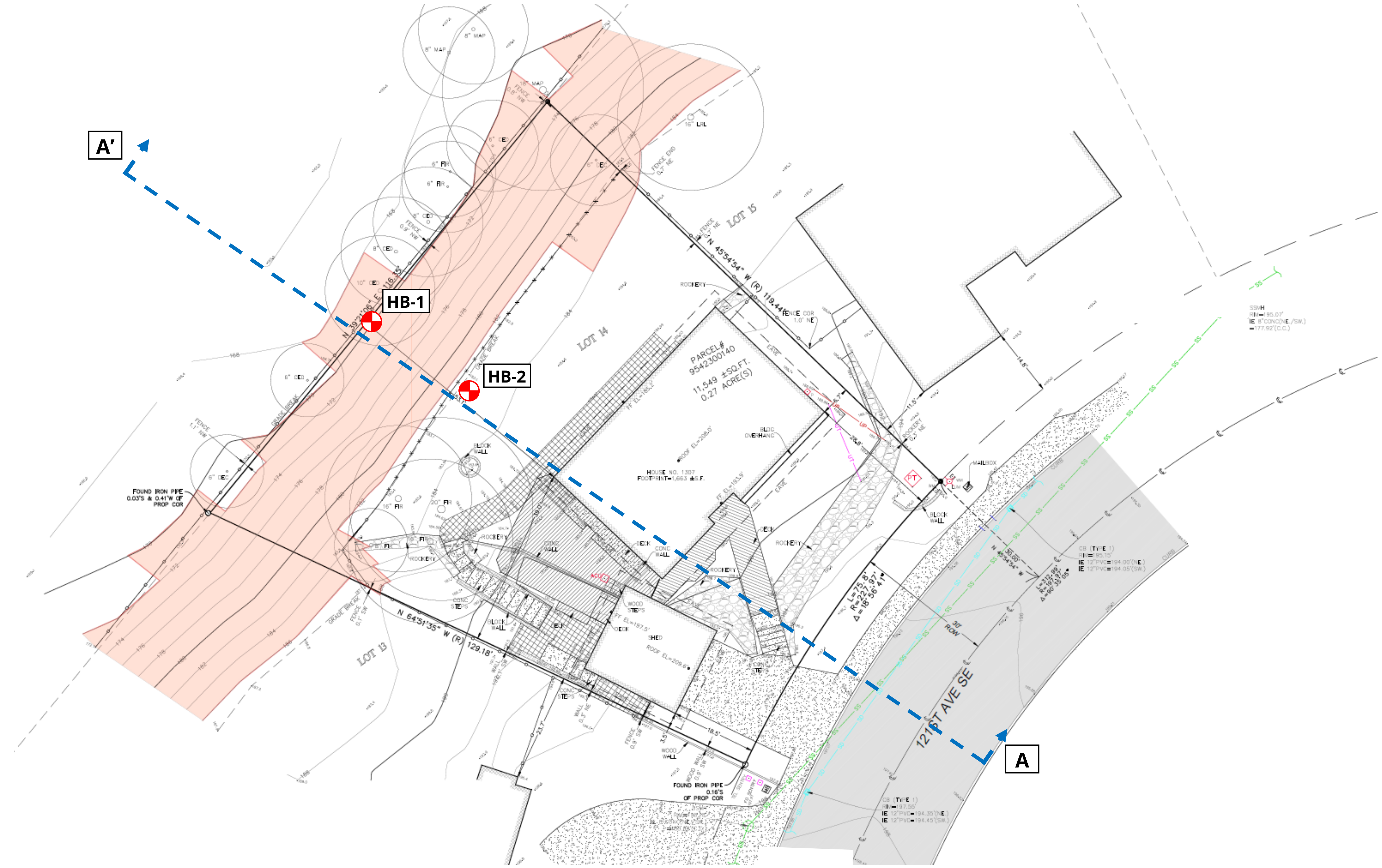
Site Vicinity Map

Proposed Accessory Structure
1307 – 121st Avenue Southeast
Bellevue, Washington
PN: 9542300140

DocID: Feldman.121stAveSE.F	December 2020	Figure 2



Topographic & Boundary Survey prepared by Terrane dated October 2, 2020

Hand boring number and approximate location

Slope stability cross section

Scale: 1" = 30'

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Site and Exploration Plan
Proposed Accessory Structure
1307 – 121st Avenue Southeast
Bellevue, Washington
PN: 9542300140

DocID: Feldman.121stAveSE.F	December, 2020	Figure 3

Approximate Site Location

Map created from GIS data provided by Web Soil Survey
(http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx)

Soil Type	Soil Name	Parent Material	Slopes	Erosion Hazard	Hydrologic Soils Group
AgC	Alderwood gravelly loam	Glacial drift or outwash over glaciomarine deposits	8 to 15	Moderate	C
AmC	Arents, Alderwood material	Basal till	6 to 15	Slight to moderate	B/D
KpD	Kitsap silt loam	Glacial lake sediment	15 to 30	Moderate	C

N W E S

Not to Scale

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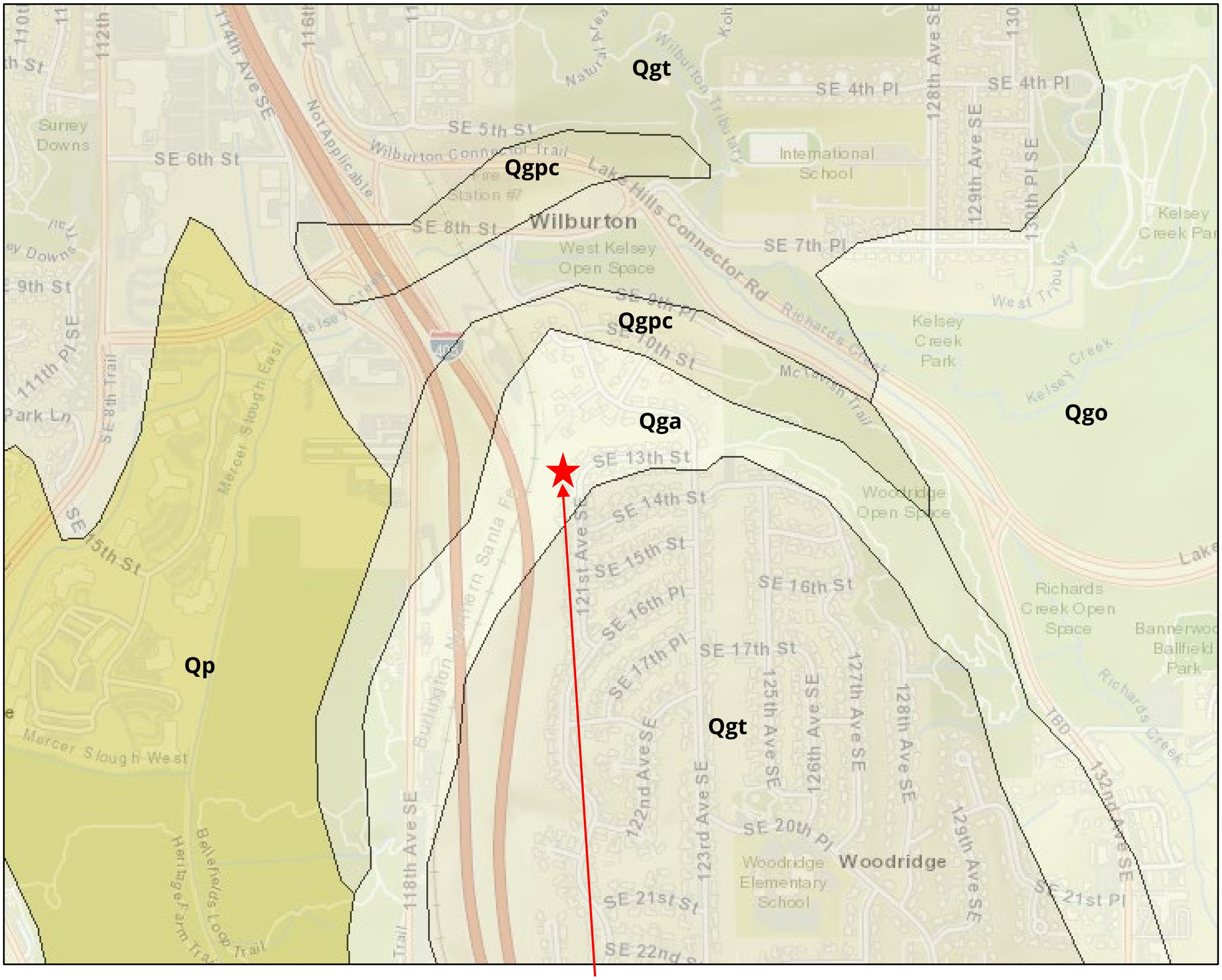
NRCS Soils Map

Proposed Accessory Structure
1307 – 121st Avenue Southeast
Bellevue, Washington
PN: 9542300140

DocID: Feldman.121stAveSE.F	December 2020	Figure 4



Approximate Site Location

Figure created from Washington State Department of Natural Resources Geology Portal

Qgt	Vashon till
Qga	Advance outwash deposits
Qgpc	Pre-Fraser continental glacial drift and nonglacial deposits

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Geologic Map

Proposed Accessory Structure
1307 – 121st Avenue Southeast
Bellevue, Washington
PN: 9542300140

DocID: Feldman.121stAveSE.F	December 2020	Figure 5

Approximate Site Location

Map created from the Washington State Department of Natural Resources (Information Portal https://geologyportal.dnr.wa.gov/)

Head Scarp and Flank
Landslide Deposits

N
W E
S

Not to Scale

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WA DNR Landslide Inventory Map

Proposed Accessory Structure
1307 – 121st Avenue Southeast
Bellevue, Washington
PN: 9542300140

DocID: Feldman.121stAveSE.F	December 2020	Figure 6

Appendix A
Subsurface Explorations

SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE GRAINED SOILS More than 50% Retained on No. 200 Sieve	GRAVEL More than 50% Of Coarse Fraction Retained on No. 4 Sieve	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND More than 50% Of Coarse Fraction Passes No. 4 Sieve	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE GRAINED SOILS More than 50% Passes No. 200 Sieve	SILT AND CLAY Liquid Limit Less than 50	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY Liquid Limit 50 or more	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1. Field classification is based on visual examination of soil in general accordance with ASTM D2488-90.
2. Soil classification using laboratory tests is based on ASTM D2487-90.
3. Description of soil density or consistency are based on interpretation of blow count data, visual appearance of soils, and or test data.

SOIL MOISTURE MODIFIERS:

Dry- Absence of moisture, dry to the touch

Moist- Damp, but no visible water

Wet- Visible free water or saturated, usually soil is obtained from below water table

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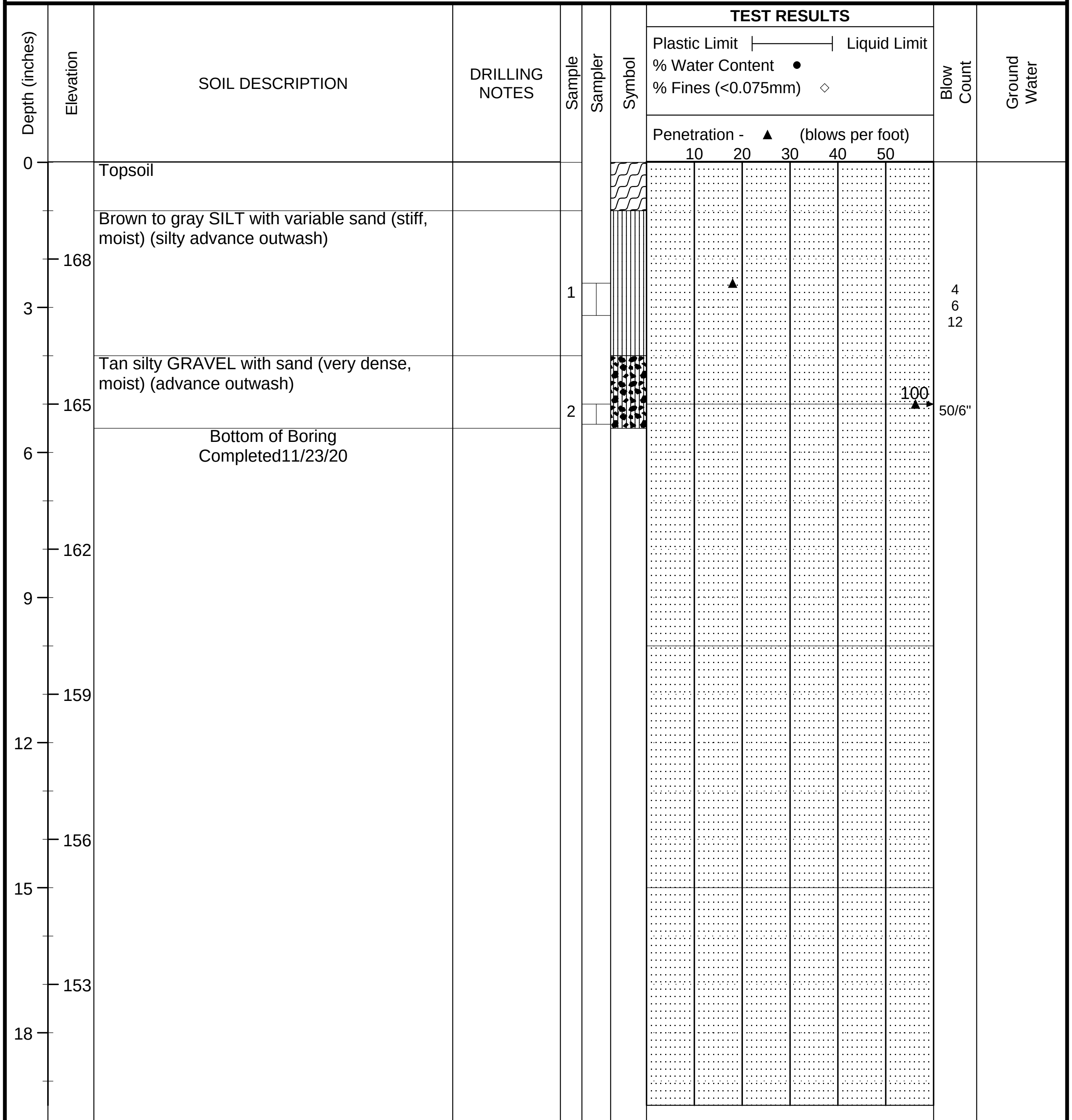
Unified Soils Classification System

Proposed Accessory Structure
1307 – 121st Avenue Southeast
Bellevue, Washington
PN: 9542300140

DocID: Feldman.121stAveSE.F	December 2020	Figure A-1

TOTAL DEPTH:	5.5	**DRILLING METHOD:**	Hand auger	**LOGGED BY:**	NAF
TOP ELEVATION:	170	**DRILLING COMPANY:**	GeoResources	**HAMMER TYPE:**	Donut
LATITUDE:		**DRILL RIG:**	Hand auger	**HAMMER WEIGHT:**	45 lbs
LONGITUDE:		**NOTES:**			



NOTES
1. Refer to log key for definition of symbols, abbreviations and codes
2. USCS designation is based on visual manual classification and selected lab testing
3. Groundwater level, if indicated, is for the date shown and may vary
4. N.E. = Not Encountered
5. ATD = At the Time of Drilling

Proposed Shed
1307 - 121st Ave SE
Bellevue, WA

LOG OF BORING HB-1

JOB: Feldman.121stAveSE *Sheet 1 of 1*

GeoResources, LLC **FIG.** A-2

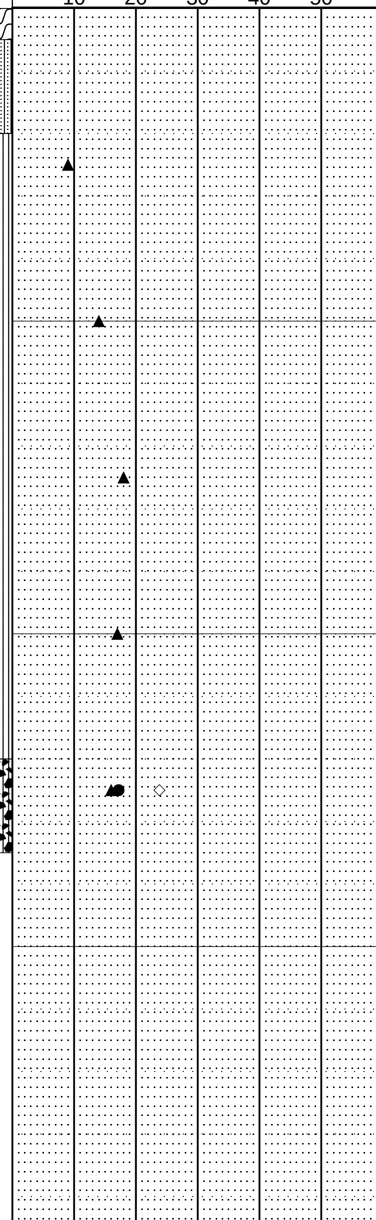
TOTAL DEPTH:	13.5	**DRILLING METHOD:**	Hand auger	**LOGGED BY:**	NAF
TOP ELEVATION:	183	**DRILLING COMPANY:**	GeoResources	**HAMMER TYPE:**	Donut
LATITUDE:		**DRILL RIG:**	Hand auger	**HAMMER WEIGHT:**	45 lbs
LONGITUDE:		**NOTES:**			

Depth (inches)	Elevation	SOIL DESCRIPTION	DRILLING NOTES	Sample	Sampler	Symbol	Blow Count	Ground Water
0	183	Topsoil						
		Tan silty SAND (medium dense, moist to wet) (fill)						
		Tan SILT with variable sand, clasts of hard gray clayey silt (stiff, moist) (fill)		1			4 3 6	
3	180							
				2			4 7 7	
6	177							
				3			5 8 10	
9	174	less than 1 inch diameter plastic debris at 9 feet						
				4			6 8 9	
12	171	Brown silty GRAVEL with sand, small organics at upper contact (medium dense, moist) (advance outwash)		5			6 10 6	
		Bottom of Boring Completed11/23/20						
15	168							
18	165							

TEST RESULTS

Plastic Limit ├──┤ Liquid Limit
% Water Content ●
% Fines (<0.075mm) ◇

Penetration - ▲ (blows per foot)
10 20 30 40 50



NOTES
1. Refer to log key for definition of symbols, abbreviations and codes
2. USCS designation is based on visual manual classification and selected lab testing
3. Groundwater level, if indicated, is for the date shown and may vary
4. N.E. = Not Encountered
5. ATD = At the Time of Drilling

Proposed Shed
1307 - 121st Ave SE
Bellevue, WA

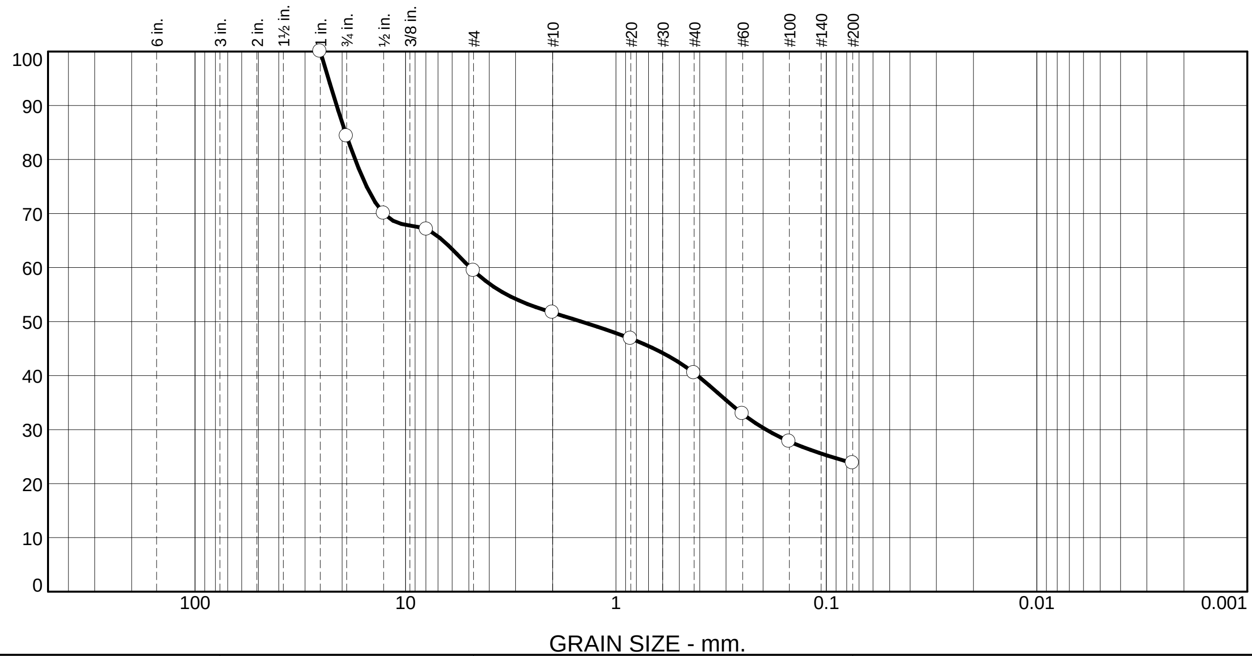
LOG OF BORING HB-2

JOB: Feldman.121stAveSE *Sheet 1 of 1*

GeoResources, LLC **FIG.** A-3

Appendix B
Laboratory Test Results

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	15.7	24.9	7.7	11.2	16.7	23.8	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1	100.0		
.75	84.3		
.5	70.0		
.3125	67.0		
#4	59.4		
#10	51.7		
#20	46.8		
#40	40.5		
#60	32.9		
#100	27.8		
#200	23.8		

* (no specification provided)

Material Description

Silty GRAVEL with sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 21.2471 D_{85}= 19.3058 D_{60}= 4.9234
D_{50}= 1.4639 D_{30}= 0.1936 D_{15}=
D_{10}= C_u= C_c=

Remarks

Moisture: 17.2%

Date Received: 12/2/20 **Date Tested:** 12/3/20
Tested By: EJF/DBS
Checked By: NAF
Title: PM

Source of Sample: HB-2 **Depth:** 12.5 **Date Sampled:** 12/1/20
Sample Number: 5

GeoResources, LLC Fife, WA	**Client:** Kyle Feldman **Project:** Proposed Shed **Project No:** Feldman.121stAveSE **Figure**

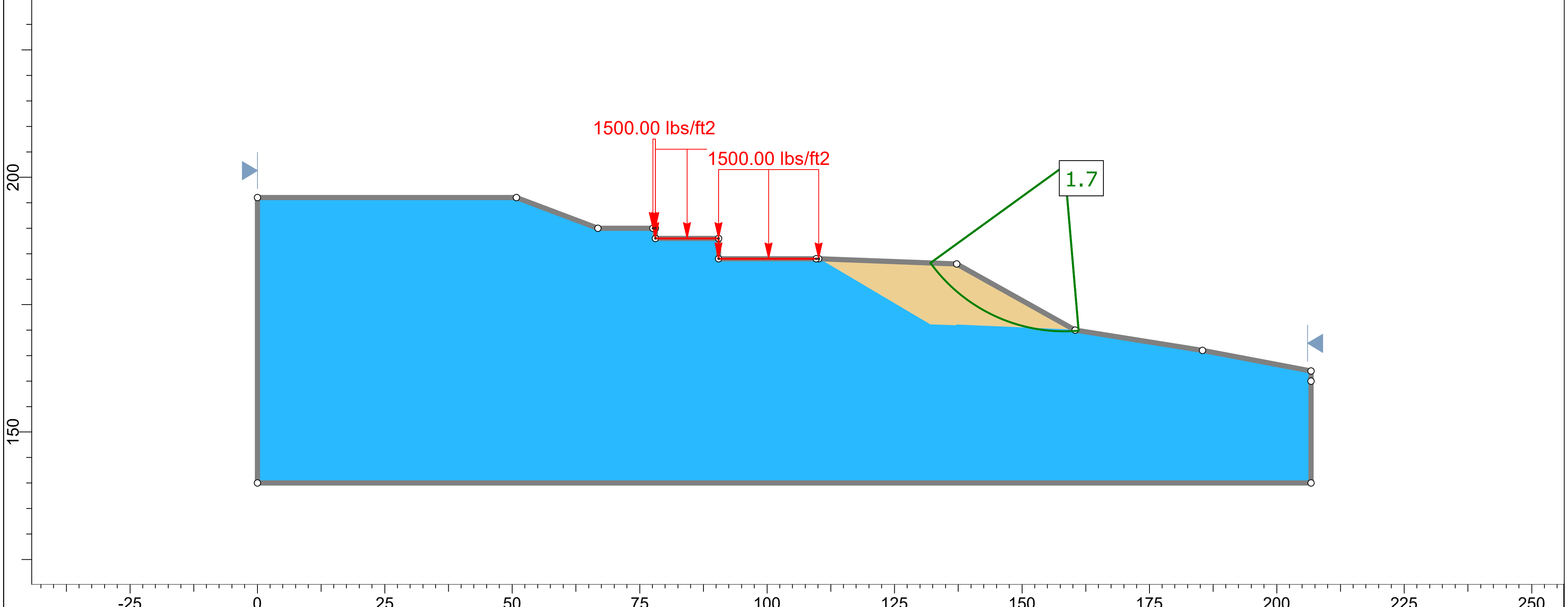
These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicitive of apparently identical samples.

Tested By: ______________ **Checked By:** ______________

Appendix C

Slope Stability Analyses

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Ru
Advance Outwash		135	Mohr-Coulomb	0	38		None	0
Fill		115	Mohr-Coulomb	150	22		None	0
Concrete		155	Undrained	14000		Constant	None	0



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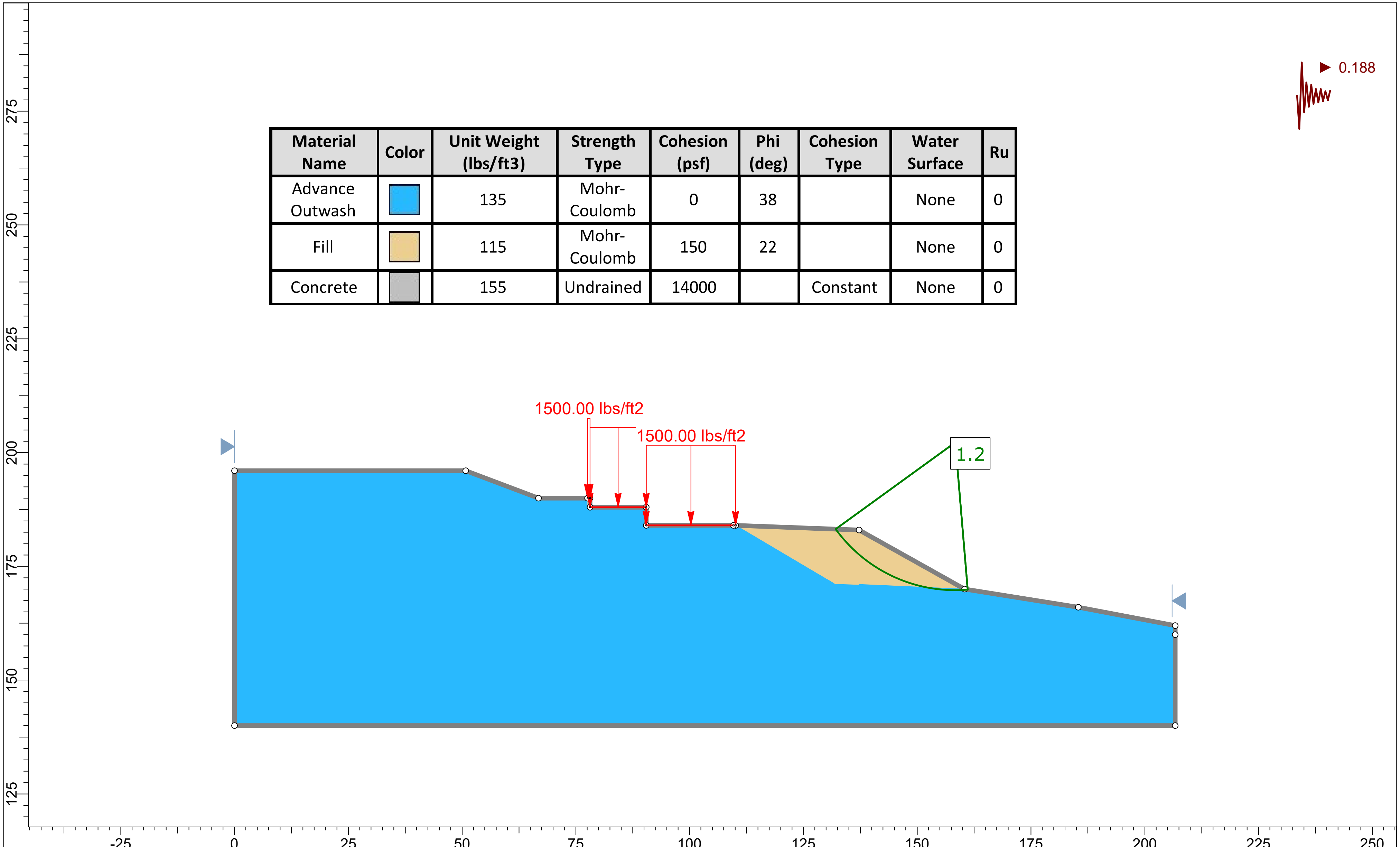
SLIDEINTERPRET 9.005

Project	Feldman.121stAveSE		
Group	Existing	*Scenario*	Master Scenario
Drawn By	NAF	*Company*	GeoResources, LLC
Date	10/7/2020, 3:05:46 PM	*File Name*	Feldman.slope.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Ru
Advance Outwash		135	Mohr-Coulomb	0	38		None	0
Fill		115	Mohr-Coulomb	150	22		None	0
Concrete		155	Undrained	14000		Constant	None	0

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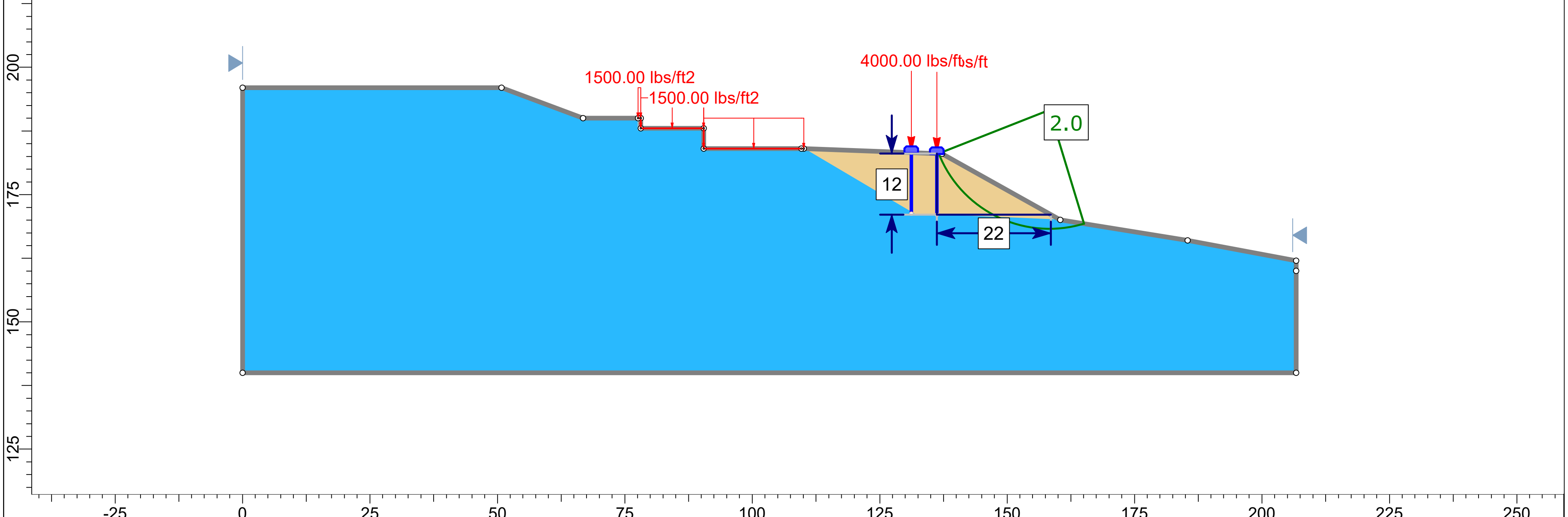
SLIDEINTERPRET 9.005

Project	Feldman.121stAveSE		
Group	Existing	*Scenario*	Seismic
Drawn By	NAF	*Company*	GeoResources, LLC
Date	10/7/2020, 3:05:46 PM	*File Name*	Feldman.slope.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Ru
Advance Outwash		135	Mohr-Coulomb	0	38		None	0
Fill		115	Mohr-Coulomb	150	22		None	0
Concrete		155	Undrained	14000		Constant	None	0

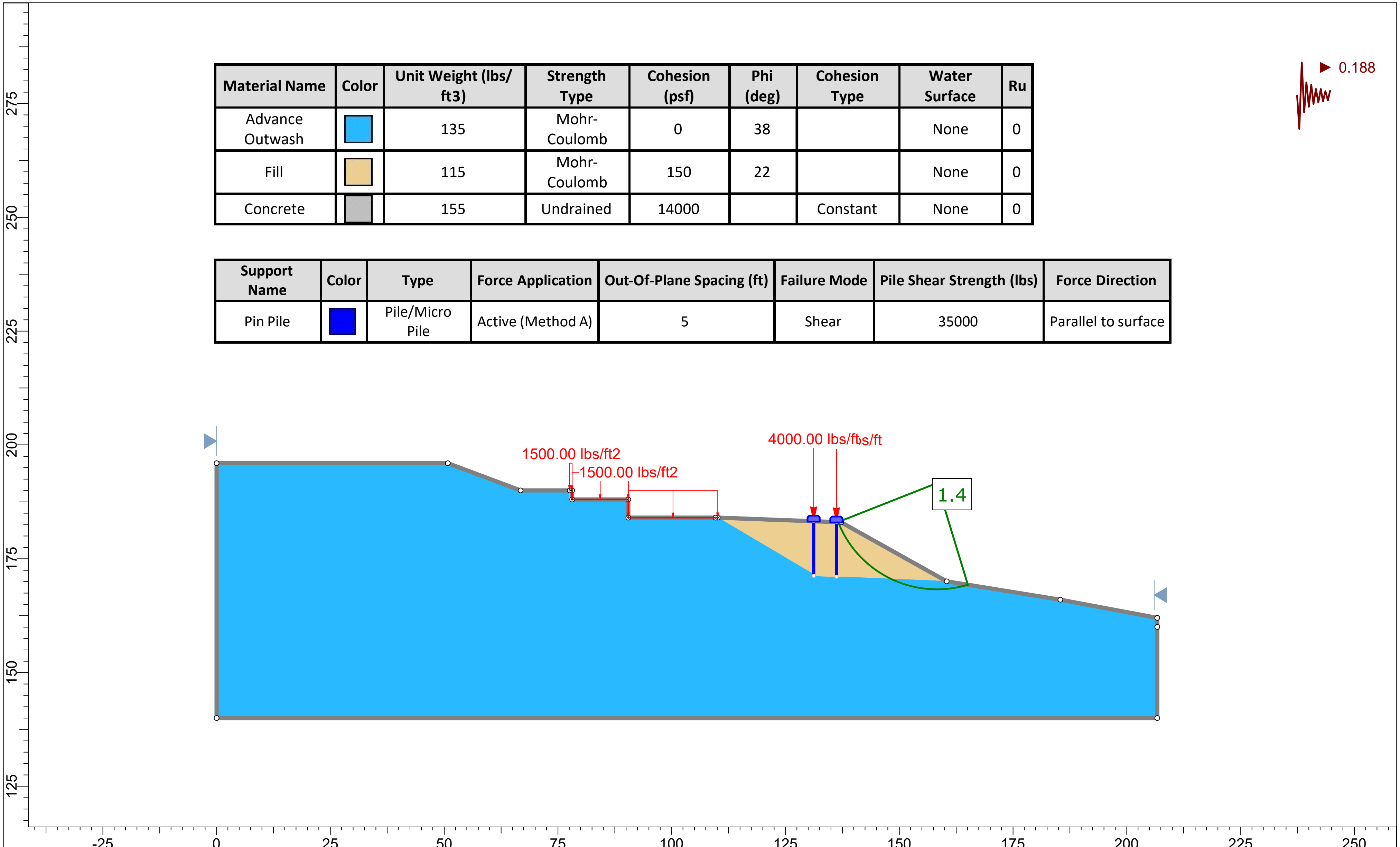
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Direction
Pin Pile		Pile/Micro Pile	Active (Method A)	5	Shear	35000	Parallel to surface



GEORESOURCES earth science & geotechnical engineering	Project	Feldman.121stAveSE	
	Group: Proposed	Scenario: Master Scenario	
	Drawn By: NAF	Company: GeoResources, LLC	
SLIDEINTERPRET 9.005	Date: 10/7/2020, 3:05:46 PM	File Name: Feldman.slope.slmd	

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Ru
Advance Outwash		135	Mohr-Coulomb	0	38		None	0
Fill		115	Mohr-Coulomb	150	22		None	0
Concrete		155	Undrained	14000		Constant	None	0

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Direction
Pin Pile		Pile/Micro Pile	Active (Method A)	5	Shear	35000	Parallel to surface



GEORESOURCES
earth science & geotechnical engineering
SLIDEINTERPRET 9.005

Project	Feldman.121stAveSE		
Group	Proposed	*Scenario*	Seismic
Drawn By	NAF	*Company*	GeoResources, LLC
Date	10/7/2020, 3:05:46 PM	*File Name*	Feldman.slope.slmd

Slide Analysis Information

Feldman.slope

Project Summary

File Name:	Feldman.slope.slmd
Slide Modeler Version:	9.006
Project Title:	Feldman.121stAveSE
Analysis:	Slope Stability
Author:	NAF
Company:	GeoResources, LLC
Date Created:	10/7/2020, 3:05:46 PM

Currently Open Scenarios

Group Name	Scenario Name	Global Minimum	Compute Time
Existing	Master Scenario	Gle/morgenstern-price: 1.742070	00h:00m:00.980s
	Seismic	Gle/morgenstern-price: 1.234830	00h:00m:01.55s
Proposed	Master Scenario	Gle/morgenstern-price: 1.984600	00h:00m:01.238s
	Seismic	Gle/morgenstern-price: 1.420310	00h:00m:01.220s

Analysis Options

<u>All Open Scenarios</u>

Slices Type:	Vertical
Analysis Methods Used	
	GLE/Morgenstern-Price with interslice force function (Half Sine)
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Surface Options

All Open Scenarios

Surface Type:	Circular
Search Method:	Slope Search
Number of Surfaces:	5000
Upper Angle [deg]:	Not Defined
Lower Angle [deg]:	Not Defined
Composite Surfaces:	Disabled
Reverse Curvature:	Invalid Surfaces
Minimum Elevation:	Not Defined
Minimum Depth [ft]:	2
Minimum Area:	Not Defined
Minimum Weight:	Not Defined

Seismic Loading

Existing - Master Scenario

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Existing - Seismic

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No
Seismic Load Coefficient (Horizontal):	0.188

Proposed - Master Scenario

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Proposed - Seismic

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No
Seismic Load Coefficient (Horizontal):	0.188

Loading

Existing

Distribution:	Constant
Magnitude [psf]:	1500
Orientation:	Vertical

Proposed

Distribution:	Constant
Magnitude [psf]:	1500
Orientation:	Vertical
Angle from horizontal [deg]:	270
Magnitude:	4000
Angle from horizontal [deg]:	270
Magnitude:	4000

Materials

Advance Outwash	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	135
Cohesion [psf]	0
Friction Angle [deg]	38
Water Surface	Assigned per scenario
Ru Value	0
Fill	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	115
Cohesion [psf]	150
Friction Angle [deg]	22
Water Surface	Assigned per scenario
Ru Value	0
Concrete	
Color	
Strength Type	Undrained
Unit Weight [lbs/ft3]	155
Cohesion [psf]	14000
Cohesion Type	Constant
Water Surface	Assigned per scenario
Ru Value	0

Materials In Use

Material	Existing	Seismic	Proposed	Seismic
Advance Outwash	✓	✓	✓	✓
Fill	✓	✓	✓	✓
Concrete	✓	✓	✓	✓

Support

Pin Pile	
Color	
Support Type	Pile/Micro Pile
Force Application	Active
Out-of-Plane Spacing [ft]	5
Failure Mode	Shear
Pile Shear Strength [lb]	35000
Force Direction	Parallel to surface

Global Minimums

Existing - Master Scenario

Method: gle/morgenstern-price

FS	1.742070
Center:	158.344, 202.247
Radius:	32.474
Left Slip Surface Endpoint:	132.050, 183.190
Right Slip Surface Endpoint:	161.090, 169.890
Resisting Moment:	317011 lb-ft
Driving Moment:	181973 lb-ft
Resisting Horizontal Force:	8570.53 lb
Driving Horizontal Force:	4919.73 lb
Total Slice Area:	118.874 ft2
Surface Horizontal Width:	29.0402 ft
Surface Average Height:	4.09342 ft

Existing - Seismic

Method: gle/morgenstern-price

FS	1.234830
Center:	158.344, 202.247
Radius:	32.474
Left Slip Surface Endpoint:	132.050, 183.190
Right Slip Surface Endpoint:	161.090, 169.890
Resisting Moment:	308318 lb-ft
Driving Moment:	249685 lb-ft
Resisting Horizontal Force:	8395.79 lb
Driving Horizontal Force:	6799.16 lb
Total Slice Area:	118.874 ft2
Surface Horizontal Width:	29.0402 ft
Surface Average Height:	4.09342 ft

Proposed - Master Scenario

Method: gle/morgenstern-price

FS	1.984600
Center:	158.264, 191.596
Radius:	23.351
Left Slip Surface Endpoint:	136.543, 183.024
Right Slip Surface Endpoint:	165.057, 169.255
Resisting Moment:	255043 lb-ft
Driving Moment:	128511 lb-ft
Resisting Horizontal Force:	9339.34 lb
Driving Horizontal Force:	4705.9 lb
Total Slice Area:	116.976 ft2
Surface Horizontal Width:	28.514 ft
Surface Average Height:	4.1024 ft

Proposed - Seismic

Method: gle/morgenstern-price

FS	1.420310
Center:	158.264, 191.596
Radius:	23.351
Left Slip Surface Endpoint:	136.543, 183.024
Right Slip Surface Endpoint:	165.057, 169.255
Resisting Moment:	248230 lb-ft
Driving Moment:	174772 lb-ft
Resisting Horizontal Force:	9185.98 lb
Driving Horizontal Force:	6467.6 lb
Total Slice Area:	116.976 ft2
Surface Horizontal Width:	28.514 ft
Surface Average Height:	4.1024 ft